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Physical modelling of soft clay consolidation using vacuum-surcharge method

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PHYSICAL MODELLING OF SOFT CLAY CONSOLIDATION USING VACUUM-SURCHARGE METHOD

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ABSTRACT

In this paper, the large-scale consolidation apparatus was employed to investigate the performance of vacuum preloading with conventional surcharge loading. Several tests were conducted to examine the effect of a vacuum and associated parameters such as smear zone and soil permeability characteristics. The measured settlement and excess pore pressure indicates that the vacuum application has certain benefits to control excess pore pressure for a given total loading application. The analytical modelling for radial consolidation and vacuum preloading considering both variation of soil compressibility and permeability was employed to predict the soil consolidation behaviour. It is shown that the analytical model can reasonably capture the laboratory behaviour.

1 INTRODUCTION

Kjellman (1952) pioneered vacuum preloading method in conjunction with vertical drains to improve soil compressibility and shear strength. An effective stress in the soil mass increases via suction pressure, while total stress remains constant (Qian *et al.,* 1992). This method can accelerate the consolidation process when additional surcharge load is applied. A system of surcharge and vacuum preloading to stabilise soft soil has been successfully applied in large highway and reclamation projects (Indraratna *et al*., 2011; Chu *et al*., 2000). Mohamedelhassan and Shang (2002) studied the application of vacuum pressure and its benefits without considering the effect of radial drainage. In this paper, a series of large-scale testing weas performed to investigate the effect of vacuum and surcharge load. Subsequently, an analytical solution for radial consolidation incorporating vacuum characteristics introduced by Indraratna *et al.* (2005) was employed to predict the vacuum consolidation responses. The advantages of vacuum-surcharge were discussed.

2 TEST APPARATUS AND SOIL PROPERTIES

Figure 1 presents a schematic diagram of the large-scale radial drainage consolidation cell built at the University of Wollongong. The main chamber consists of two half circular sections (450 mm inside diameter by 950 mm high) which can be bolted together. The cell can be assembled on a steel base. To minimise the friction along the cell boundary, a 1.5 mm thick, ultra smooth Teflon sheet (friction coefficient <0.03) was inserted around the internal circumference. The surcharge loading system with a maximum capacity of 1.2 MN can be applied via an air compressor system on a 50mm thick rigid piston, while a vacuum application system with a maximum capacity of 90 kPa was applied through a central hole of the rigid piston. The instrumentation, including a displacement and miniature pore pressure transducers, were installed to record settlement and pore pressure during consolidation. Prior to the installation, the porous stone tips of the pore pressure transducers were saturated under vacuum. The cell can also be equipped with a specially designed mandrel installation machine, which enables a prefabricated vertical drain (PVDs) to be inserted vertically along the central axis of the cell.

The total volume of soil required for each sample was approximately 0.14 m^3. Reconstituted commercial clay was used due to the difficulty in obtaining large size undisturbed samples. The basic geotechnical properties of a typical specimen are shown in Table 1. Four series of oedometer tests were conducted using reconstituted commercial Kaolin clay in order to obtain the soil compressibility and void ratio-permeability relationship. The e-logσ' and e-logk_h relationships are illustrated by Figure 2. Accordingly, the slope of the e-logσ' line (C_c) and the slope of e-logk_h line (C_k) were determined to be 0.29 and 0.45, respectively. Therefore, the corresponding C_c/C_k can be calculated as 0.64. The undrained shear strength was measured using standard consolidated undrained triaxial test.

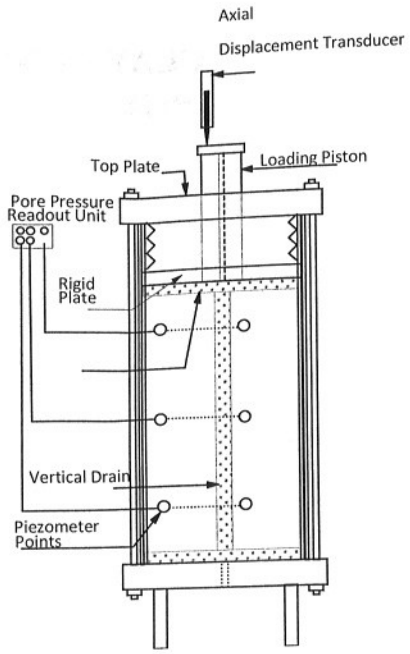


Figure 1: Large-scale consolidation apparatus.

Table 1. Soil properties of the reconstituted commercial Kaolin clay sample.

Clay content (%)	40-50
Silt Content (%)	45-60
Water content, w (%)	38
Liquid limit, w_L (%)	57
Plastic limit, w_p (%)	17
Unit weight (kN/m^3)	18.1
Specific gravity, G_s	2.65
Undrained shear strength (kPa)	7.3

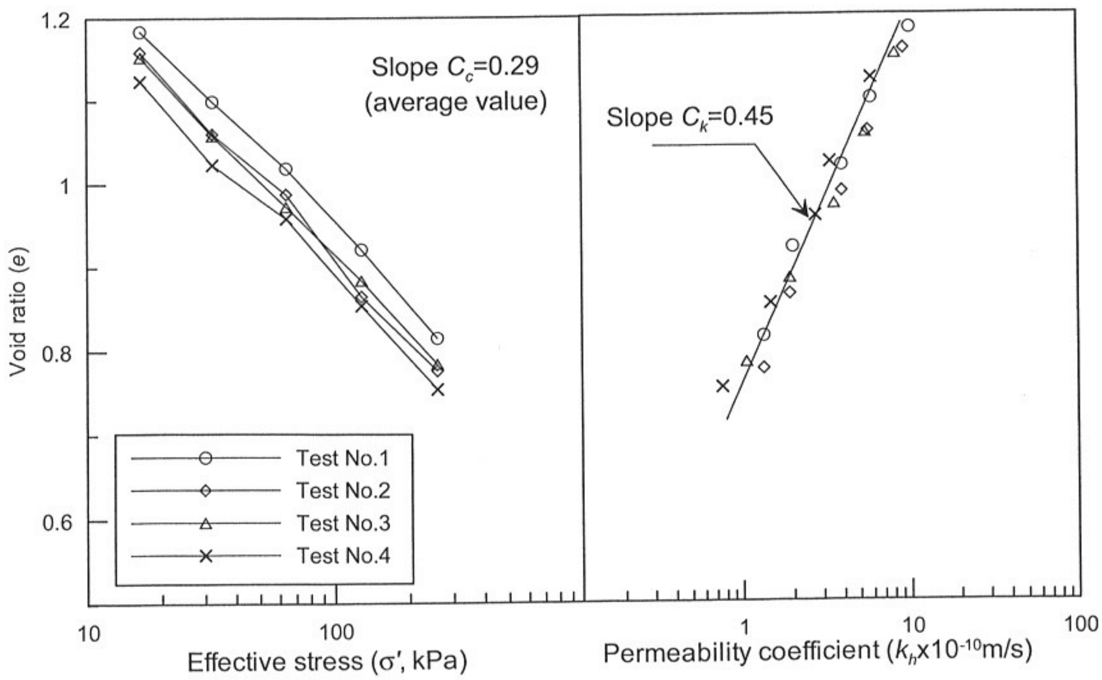


Figure 2: Typical e-logσ' and e-logk_h plots for Moruya clay (Indraratna *et al.*, 2005).

3 TESTING METHODOLOGY

In order to study the performance of vacuum preloading, three series of large-scale radial consolidation tests, including surcharge preloading alone (SP1), vacuum preloading alone (VP1), and combined surcharge and vacuum preloading (SV1) were carried out. These test procedures can generally be divided into three main stages: (a) preparation of reconstituted clay, (b) drain installation and (c) collection of oedometer sample. Table 2 summarizes the details of vacuum and surcharge applied to these test series.

Table 2: Summary of the large–scale test series.

Series	Test No.	Applied vacuum pressure (kPa)	Applied surcharge pressure (kPa)	Preconsolidation pressure (kPa)
1	SP1	0	30	20
2	VP1	20	0	20
3	SV1	20	30	20

The procedures for reconstituted clay preparation were similar for all tests. The clay was thoroughly mixed into slurry using distilled water to minimize air entrapment and provide homogeneous mix. In this study the water content of the reconstituted clay was approximately 58%. Subsequently the saturated mix was placed into 5 sub-layers (about 200 mm thick) in the cell. To avoid air trap, each layer was subjected to a vacuum and mild vibration for an hour. After the final layer was reached, a filter was placed on the surface so that any excess water bleeding out could be removed without disturbing the soil sample.

An initial consolidation pressure of 20 kPa was applied to the reconstituted clay sample via the top air pressure chamber. A vertical band drain was installed using a specially designed rectangular mandrel installation machine equipped with a mandrel guide. To avoid drain unsaturation, the PVD was submerged under water before installation. Six pore pressure transducers were inserted manually at various locations, as shown in Figure 3. Preloading and vacuum pressure was applied, as summarized in Table 2. It is noted that in order to measure the initial distribution of vacuum along the drain, pore pressure transducers T1, T2 and T3 were initially installed at the drain boundary and then moved to the location 70 mm away from the drain.

After surcharge and/or vacuum application, undisturbed vertical and horizontal specimens were collected from different locations. A procedure for collecting samples is described by Indraratna and Redana (1998). Oedometer tests were then conducted to establish the extent of smear zone. In each test series the water content was measured at 0.24 m, 0.48 m and 0.72 m from the bottom of the drain to compare with the initial condition.

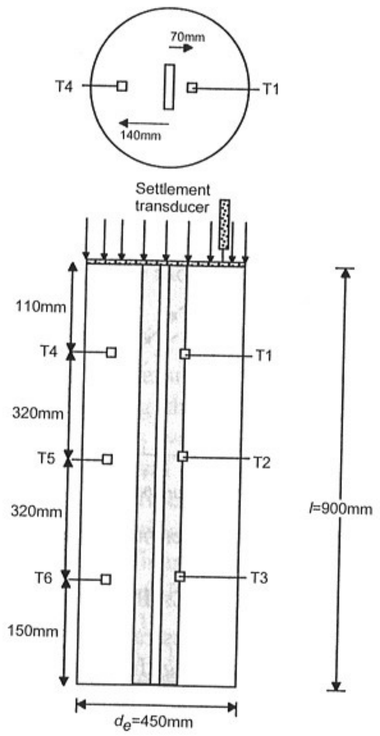


Figure 3: Schematic section of the large-scale, radial drainage consolidometer showing the central drain, associated smear zone and typical locations of pore pressure transducers.

4 ANALYTICAL SOLUTION FOR RADIAL CONSOLIDATION CONSIDERING SURCHARGE AND VACUUM PRELOADING

The dissipation rate of average excess pore pressure ratio $(R_u = \bar{u}_t / \Delta p)$ at any time factor (T_h) can be expressed as (Rujikiatkamjorn 2005):

$$R_u = (1 + p_0(1+k_1)/2\Delta p)\exp(-8T_h^* / \mu) - p_0(1+k_1)/2\Delta p \quad [1]$$

In the above expression,

$$T_h^* = P_{av} T_h \quad [2]$$

$$P_{av} = 0.5\left[1 + \left(1 + \Delta p/\sigma'_i + p_0(1+k_1)/2\sigma'_i\right)^{1-C_c/C_k}\right] \quad [3]$$

$$T_h = c_h t / d_e^2 \quad [4]$$

where, σ'_i= initial effective stress, p_0= vacuum pressure, $n = d_e/d_w$, $s = d_s/d_w$, d_e = equivalent diameter of cylinder of soil around drain, d_s = diameter of smear zone and d_w = diameter of drain well, k_h = average horizontal permeability in the undisturbed zone (m/s), and k'_h = average horizontal permeability in the smear zone (m/s). Δp = preloading pressure, T_h is the dimensionless time factor for consolidation due to radial drainage, k_1 = ratio of the vacuum pressure measured at the top and the bottom of the drain, and μ = a group of parameters representing the geometry of the vertical drain system and smear effect. Hansbo (1981) assumed the smear zone to have a reduced horizontal permeability that is constant throughout this zone. The μ parameter can be given by:

$$\mu = \ln n/s + k_h / k'_h \ln s - 0.75 \quad [5]$$

The average degree of consolidation based on excess pore pressure can be obtained as follows:

$$U_p = 1 - R_u \quad (6)$$

5 TEST RESULTS AND ANALYSIS

5.1 VACUUM PRESSURE DISTRIBUTION ALONG THE DRAIN INTERFACE

In order to obtain the vacuum pressure distribution at the drain boundary, pore pressure transducers T1, T2 and T3 were initially located close to the drain boundary for Test series SP1 and SV1. Figure 4 presents the pore pressure measurement along the drain length. It was observed that the vacuum pressure propagates instantly and also decreases along the length of the drain. The rate of vacuum pressure loss within the drain may depend on the length, the installation technique and type of PVD (core and filter properties). The observed pore pressure distributions were adopted as a boundary condition in the proposed analytical solutions and k_1 was determined to be 0.85.

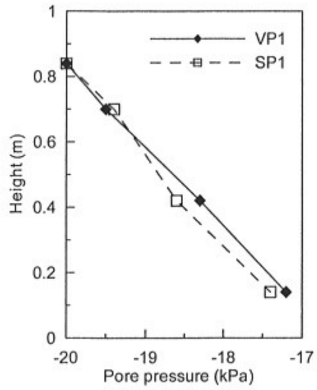
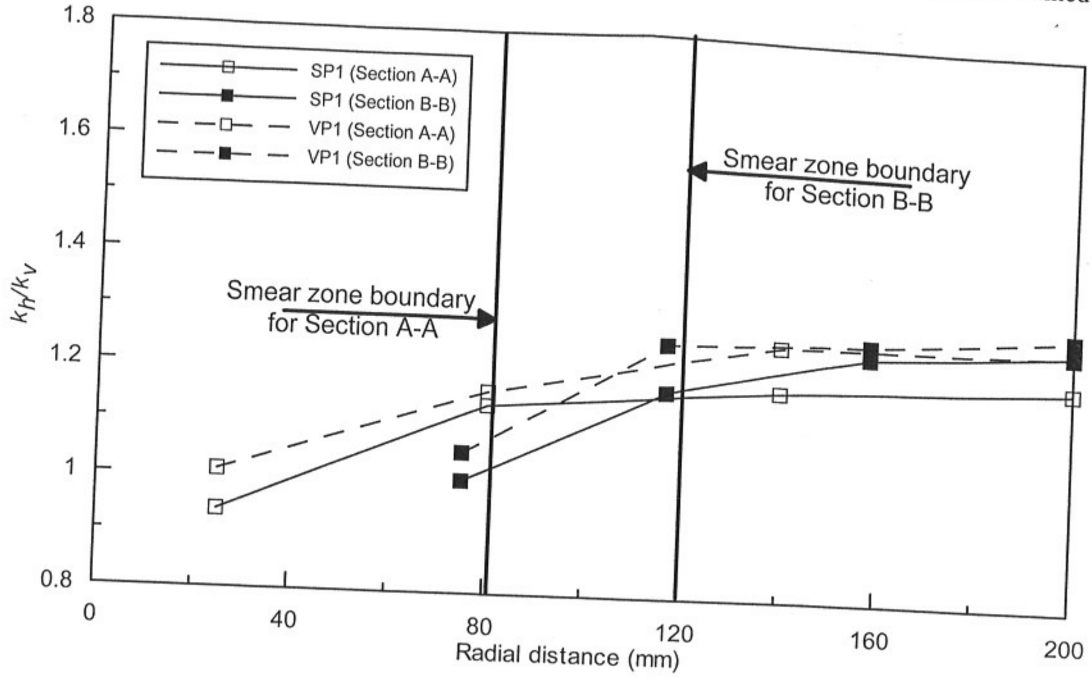


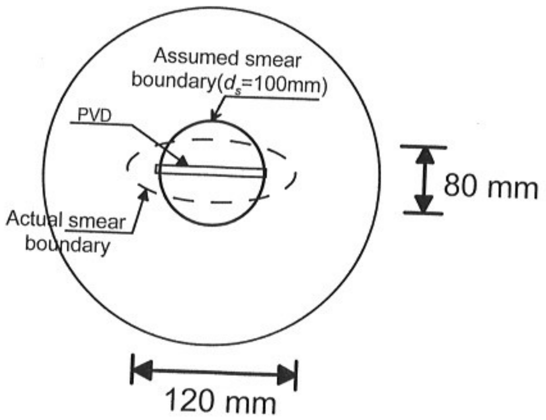
Figure 4: Distributions of measured negative pore water pressure along drain boundary in laboratory testing

5.2 EVALUATION OF THE EXTENT OF THE SMEAR ZONE

Indraratna and Redana (1998) suggested that a dimensionless ratio k_h/k_v is a proper method for determining the smear zone because it reduces the error occurring when determining the coefficient of permeability in any direction. Figure 5 illustrates the ratio k_h/k_v along the radii of the unit cell and smear zone boundary determined at the preconsolidation pressure of 40 kPa by square root fitting method. The average radius of the smear zone based on an equivalent area was 100 mm or about 3 times the value of r_w, which agrees with the observation of Indraratna and Redana (1998) for the same soil. The ratio of average permeability in undisturbed zone to smear zone (k_h/k_s) of 1.3 can be obtained from both tests.



(a)



(b)

Figure 5: Smear zone extent (a) ratio of k_h/k_v along the radial distance from central drain and (b) smear zone boundary

5.3 EXCESS PORE PRESSURE DISSIPATION

Figures 6-8 represent a comparison of excess pore pressures between the prediction and measurement. The measured excess pore pressures were calculated by subtracting the hydrostatic pore water pressure from the mesaured pore pressure, whereas the predicted excess pore pressures were determined using Equation (1). It can be seen that these excess pore pressures agree with the proposed analytical results when the correct boundary conditions were used. The dissipation rate of excess pore pressures near the impermeable boundary (e.g. T1, T2 and T3) were slower than those in the vicinity of the drain (e.g. T4, T5 and T6). At the initial state (time less than 1 day), the excess pore pressures at the vicinity of the drain (T1, T2 and T3) dropped significantly due to the free drainage boundary created by PVD. As expected, the tests simulating the surcharge loading (SP1 and SV1) experienced a much higher initial excess pore pressure compared to those with only a vacuum (VP1). In the case of a vacuum application, the final pore pressure approached the vacuum pressure at the drain boundary. The dissipation rate of excess pore water pressure depends on the magnitude of applied vacuum pressure. It is clear that the PVD allows for negative pore pressure generated along the boundary of the drain. With a vacuum applied the maximum excess pore pressure at the initial stage of embankment construction can be reduced to minimize the risk of undrained shear failure.

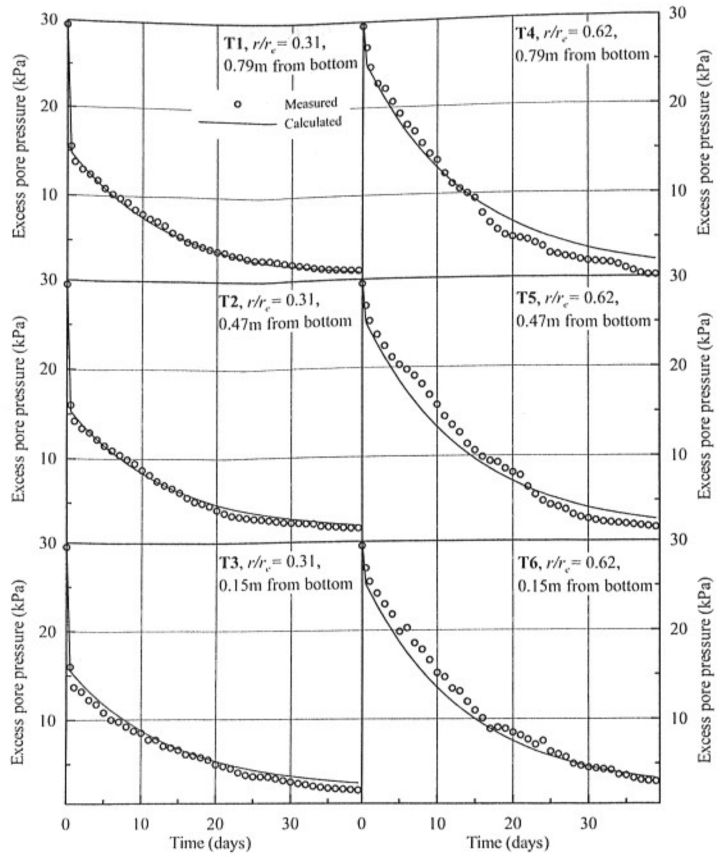


Figure 6: Comparison between the measured and calculated excess pore pressure dissipation for SP1.

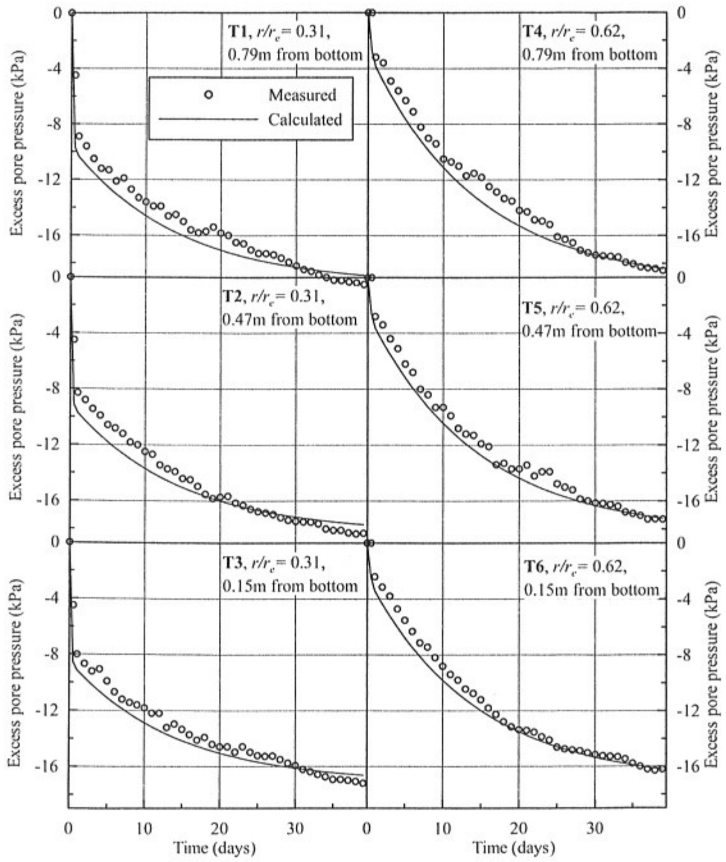


Figure 7: Comparison between the measured and calculated excess pore pressure dissipation for VP.

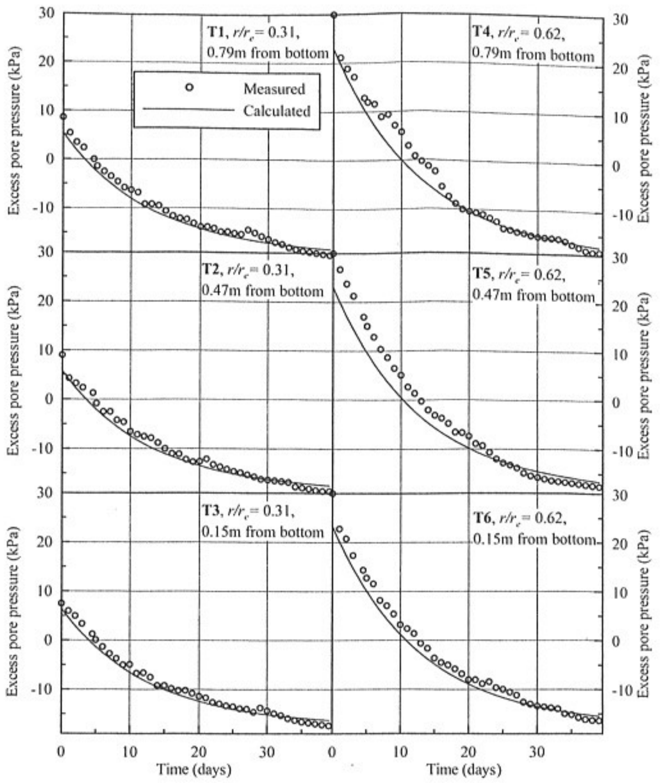


Figure 8: Comparison between the measured and calculated excess pore pressure dissipation for SV1.

5.4 CONSOLIDATION SETTLEMENT

The total settlement-time curves associated with vacuum and surcharge load are shown in Figure 9. The final degree of consolidation for each test is approximately 95%. Clearly, the accelerated consolidation increases the rate of settlement, which is analogous to increasing the applied surcharge load. However, it is also found that the measured settlement is not directly proportional to the amount of applied load, due to non-linear soil stiffness. The solution incorporates the compression indices and the variation of lateral permeability of the soils, whereas the conventional solution (Hansbo, 1981) considers constant compressibility and constant horizontal permeability. The settlement predictions from the conventional solution underestimate the laboratory results, whereas the predictions using the proposed solutions provide acceptable predictions with the laboratory data because consolidation depends on the variation of soil compressibility and permeability apart from drain configurations.

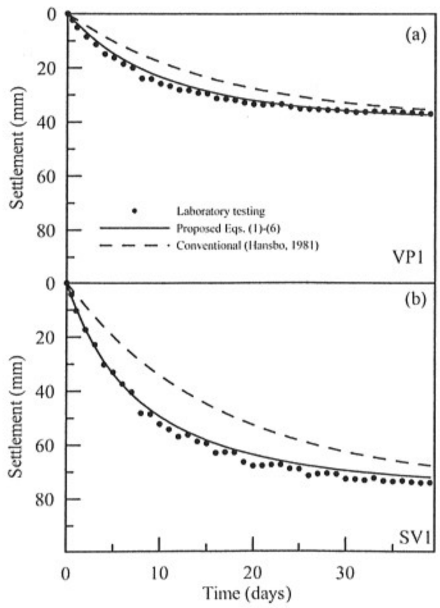


Figure 9: Settlement-time curves.

6 CONCLUSIONS

The results clearly indicate that a system of prefabricated vertical drains (PVD) combined with vacuum preloading is an effective method for consolidating soft soil. An increase in rate of settlement and pore pressure is expected when additional load is applied. The assumption of a linear variation of vacuum pressure along the drain length can be verified using the pore pressure transducers at the drain boundary located at different depth. In the field, the applied vacuum pressure at the top of the drain may not always propagate towards the bottom.

It was found that the average equivalent radius of the smear zone was approximately 100 mm, or about 3 times the value of equivalent drain radius. The average horizontal soil permeability in the undisturbed zone is 1.3 times that in the smear zone. The excess pore pressure close to the drain decreased significantly during the initial stage of vacuum application. The excess pore pressures at the end of primary consolidation approach the value of applied vacuum pressure at the boundary.

The amount of settlement observed in the tests depends on the magnitude of applied surcharge and vacuum. However, it is not linearly proportional to the magnitude of applied surcharge and vacuum due to the non-linear relationship between soil compressibility and effective stress. The settlements and excess pore water pressures were analysed using the proposed analytical solution and compared with the laboratory data. The proposed solution incorporating the compressibility indices (C_c and C_r), and the variation of horizontal permeability coefficient (k_h) gives a greater accuracy of the predictions than the previous solutions proposed by Hansbo (1981).

7 ACKNOWLEDGEMENTS

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